# REINFORCED CONCRETE RAILWAY TRESTLE

BY

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ARMOUR INSTITUTE OF TECHNOLOGY

1912

THESIS

DESIGN OF A STANDARD

REINFORCED CONCRETE

RAILWAY TRESTLE

Presented by

Thomas F. [illegible]



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Prof Civil Engineering

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Dean of Cult. Studies.

# DESIGN OF A STANDARD REINFORCED CONCRETE RAILWAY TRESTLE.

PREPARED AS A

GRADUATION THESIS

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# THE DESIGN OF A STANDARD REINFORCED CONCRETE RAILWAY TRESTLE

In railway construction numerous places are encountered where fills are prohibitive because of the nature of the ground, and where long span bridges would be too expensive to use. This is true of boggy land, swamps, river bottoms, and such places where there is no flow of water to speak of, but where fills would be undesirable. In these locations, since there is neither floating ice nor drifting material to look out for, there is no objection to the large number of supports necessary in trestles. For these reasons trestles have been used since the earliest days of railroad construction. Up to comparatively recent years wood was used exclusively for this class of structure, but it is now being replaced by steel and concrete.

For extremely high structures steel is used, while for all other structures concrete is most adaptable. The general design of the concrete structure follows closely the general lines of the old wooden one. Spans of from fifteen to twenty feet are used, supported either on piers, or on pile bents. The pile bents used, are practically duplicates of the wooden ones, but are entirely of reinforced concrete; they are used where the height of the structure is not so great as to cause excessive bending in the supports. In cases where bending does take place the piers are substitut-

ed for the pile bents. The use of the [illegible] it more economical to use larger spans than are used with pile bents, so in the following work a twenty-two foot slab has been designed for this type. The great advantage of the pile bent type is the fact that it reduces the field work to a minimum, since both the slabs and piles are cast at some central point, leaving only the pile caps to be cast at the bridge site. In the following work the complete figures are shown for the design of each part of the trestle as well as the drawings of each part and of the entire structure. The last plate shows in detail the material necessary for the various parts of the trestle.

Data

LOADING:-Live load--Cooper's E 50

Dead load--Weight of slab, ballast, and track.

Concrete = 150# per cu. ft.

Ballast = 100# " " "

Track = 150# " lineal ft.

Timber = 4 1/2# " board ft.

SPECIFICATIONS:-American Railway Engineering and Maintenance of Way Association.

SPANS:- 15, 16, 17 and 20 feet.

Considering a depth of [illegible] of ballast under the tie and a tie [illegible], the dead weight per foot of bridge is as follows:

| | |
|---|---|
| Wt. of 1 tie | = [illegible] |
| " " [illegible] cu. ft. of ballast | = [illegible]00# |
| " " 1 Ft. of track | = 150# |
| Total wt. per ft. | = [illegible]# |

Since the width of slab is 14'-0" the dead load per sq. ft. of bridge = [illegible] ÷ 14 = [illegible]

STRESSES IN SLABS

A.-15 ft. span:- Live load bending moment [illegible] = 1,[illegible],[illegible] inch pounds.

This moment is distributed over [illegible] feet of slab, therefore the bending moment taken by 1 foot = $\frac{1,[illegible],[illegible]}{7}$ = [illegible] in. lbs.

Assume weight of slab to be 300 lbs. per sq. ft.

Dead load bending moment = $\frac{3[illegible]}{8}$ = 1[illegible],[illegible] in. lbs.

Impact=[illegible] $\frac{[illegible]}{[illegible]+1[illegible]}$ =[illegible],000 in. lbs.

Total bending moment =[illegible],100 in. lbs.

Maximum live load shear [illegible] =[illegible],000

Impact [illegible] =[illegible],000 $\frac{[illegible]}{[illegible]}$ =[illegible]

Dead load shear per foot of width [illegible]

In the following calculation the following symbols will be used.

$M_1$=Live load bending moment per rail

$I$ =Impact " " " "

$M_d$=Dead load " "

$R_1$=Live load shear " "

$I_s$=Impact " "

$R_d$=Dead load " "

$M$ =Total bending moment

$S$ =Total shear

$M_1'$, $I'$, $M_d'$, $R_1'$, $I_s'$ and $R_d'$, [illegible] per foot of width.

B.-16ft. span:- $M_1$=[illegible],100,000 in lbs.

$I$ =[illegible],000,000 in lbs.

$M_1'$=[illegible]00,000 in lbs.

$I'$ =[illegible] in lbs.

$M_d'$=$\frac{[illegible]\times 1[illegible]^2\times 12}{8}$ =150,000 in. lbs.

$M'$=[illegible] in. lbs.

$R_1$=[illegible]

$R_1'$=[illegible]

$I_s'$=[illegible]

$R_d'$=[illegible]

$S'$ =$R_d'$+$I_s'$+$R_1'$=1[illegible]

C.--17 ft. SPAN:- $M_1$=[illegible],[illegible]8[illegible],000 in. lbs.

$M_1'$=[illegible]9[illegible],[illegible]00 in. lbs.

I' =[illegible]13,500 in. lbs.

$M_d'$=1[illegible]9,000 in. lbs.

M' =[illegible]13,[illegible]00 in lbs.

$R_1$ = [illegible],[illegible]00 #

$R_1'$= [illegible],[illegible]00#

$I_s'$= [illegible],[illegible]0#

$R_d'$= [illegible],[illegible]10#

S '= 1[illegible],[illegible]0#

D.--[illegible]ft. SPAN:- Assume thickness of slab of [illegible]ft. [illegible] in.

$M_1$= [illegible],[illegible]90,000 in lbs.

$M_1'$=5[illegible]7,1[illegible]0 in. lbs.

I'=4[illegible],000 in. lbs.

$M_d'$=[illegible]6,000 in. lbs.

M'=1,[illegible],1[illegible]0 in. lbs.

$R_1$= [illegible],[illegible]0[illegible]#

$R_1'$=[illegible],450#

$I_s'$=[illegible],[illegible]00

$R_d'$=5,[illegible]0#

S'=[illegible],[illegible]0#

DESIGN OF SLABS

In all the following work the symbols used by Turneaure and Maurer are used.

n=15 fs=15000# fc=[illegible]#

$$\frac{fs}{nfc} = \frac{1-k}{k}$$

$$\frac{15000}{15x750} = \frac{1-k}{k} = 1.333$$

$$K = .428$$

$$J = 1-1/3k = .857$$

$$M_s = F_s \; Adj$$

$$A = \frac{Ms}{Fs \; jd}$$

$$Mc = 1/2Fc \; Kj \; bd^2$$

$$Fc = \frac{2Mc}{Kj \; bd^2}$$

A.--15ft. SLAB:--D.=23-2 1/2=20 1/2 in.

$M = 656,100$ in. lbs.

$$A = \frac{656,100}{15000x \; .857x20.5} = 2.50 \text{ sq. in.}$$

$$Fc = \frac{2x656,100}{.428x.857x12x20.5^2} = 706\#$$

Use 7/8 sq. in. bars spaced 3 1/2 centers.

B.--16ft. SLAB:--D.=24-2 1/2=21 1/2 in.

$M - 735,700$ in.lbs.

$$A = \frac{735,700}{15000x.857x21.5} = 2.66$$

$$Fc - \frac{2x735,700}{.428x.857x12x21.5^2} = 722\#$$

Use 7/8 in. sq. bars at 3 1/2 in. cents.

C.--17ft. SLAB:--D. =25-2 1/2=22 1/2 in.

$M = 813,900$ in. lbs.

$$A = \frac{813,900}{15000x.857x22.5} = 2.82 \text{ sq. in.}$$

$$Fc = \frac{2x813,900}{.428x857x12x22.5^2} = 729\#$$

Use 7/8 in. sq. at 3" centers.

D.--22ft. SLAB:- D=3[illegible]-3=2[illegible] in.

$$M=1,775,150 \text{ in. lbs.}$$

$$A=\frac{1,775,150}{15000x.875x[illegible]} = 5.[illegible] \text{ sq. in.}$$

$$Fc=\frac{2x1,775,15[illegible]}{.42[illegible]x[illegible]7x12x[illegible]} = 74[illegible]\#$$

Use 1 in. round bars at 2 1/2 in. centers.

The following diagrams show where rods should be turned up to resist shear. No stirrups are needed.

LIFTING STIRRUPS

$$\text{Weight of 22 foot slab}=\frac{32}{12}x7x22x150=61,700\#$$

$$\frac{61,700}{15000}=4.1 \text{ sq. in.} = [illegible] \text{ of stirrup.}$$

Use 2 stirrups 1 5/8 in. round.

$$\text{Weight of 1-17 foot slab}=\frac{25}{12}x7x17x150=37,300$$

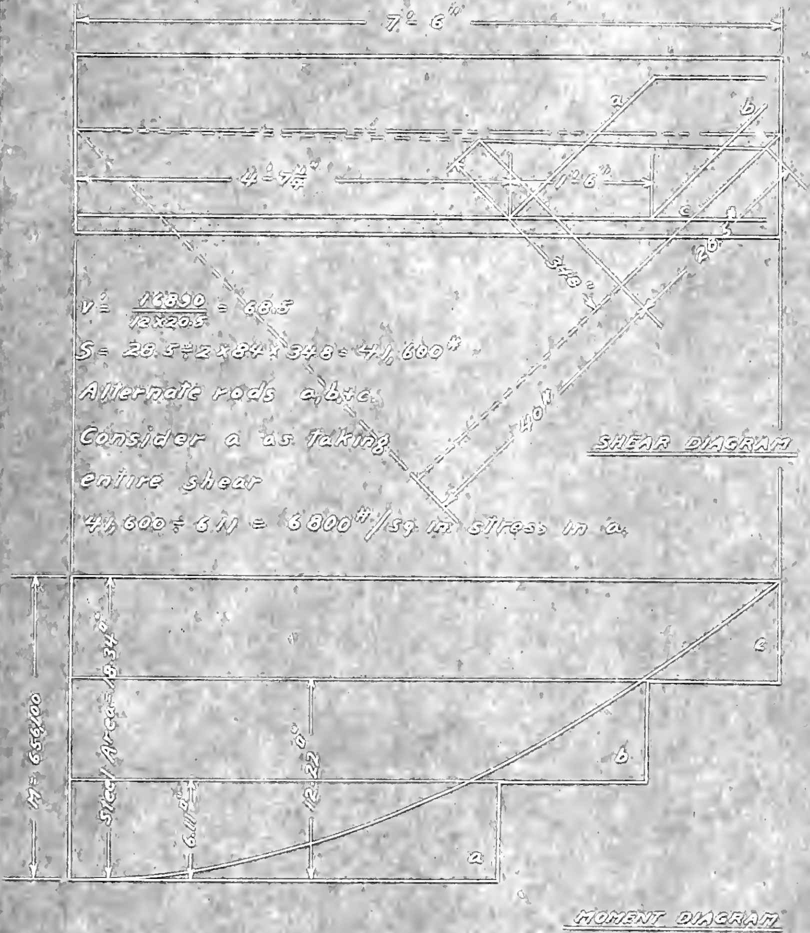
$$\frac{37,300}{15,000}=2.49 \text{ sq. in.}$$

Use 2 stirrups 1 1/2 in. round.

# 15'-0" SLAB

WIDTH 7'-0"  DEPTH 23"  DEPTH TO STEEL $20\frac{1}{2}$"

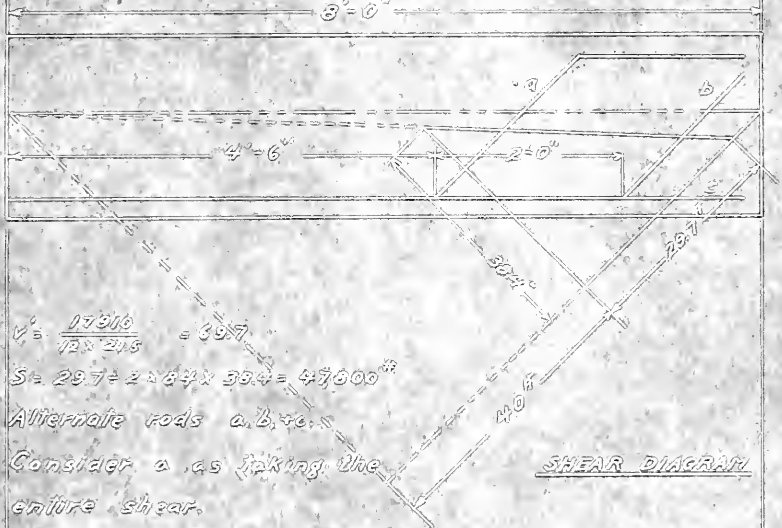
STEEL $\frac{7}{8}$" □ BARS  $3\frac{1}{2}$" C. TO C.



# 16'-0" SLAB

WIDTH 7'-0" DEPTH 24" DEPTH TO STEEL 21½"

STEEL ⅞" ⌀ BARS 3½" C to C




$v' = \frac{17910}{12 \times 21.5} = 69.7$

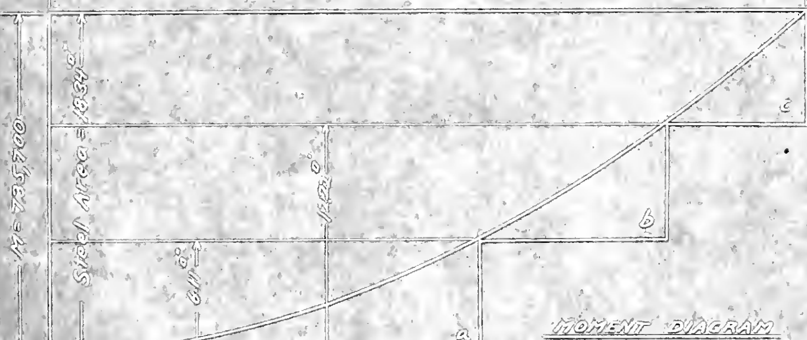
$S = 29.7 \div 2 \times 84 \times 38.4 = 47800$#

Alternate rods a, b, + c.

Consider a as taking the entire shear.

47800 ÷ 6.11 = 7850#/sq in. stress in a.

SHEAR DIAGRAM

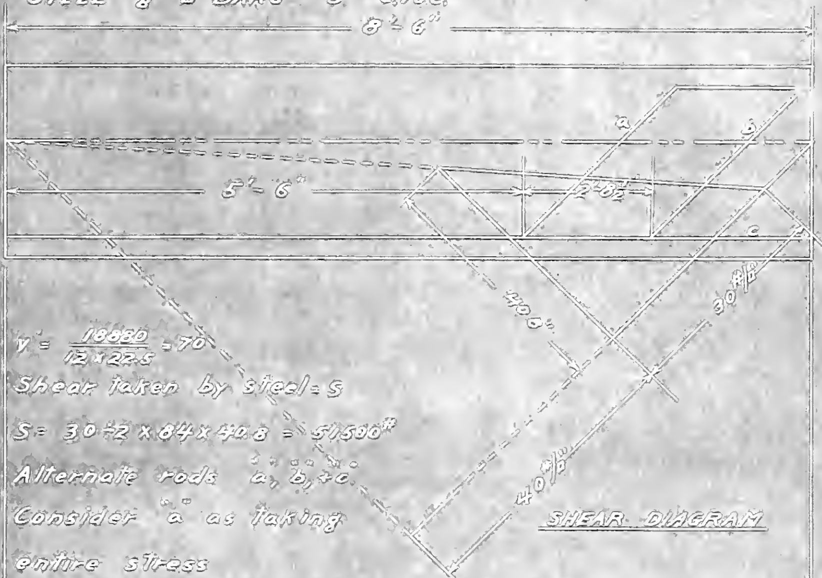



MOMENT DIAGRAM

# 17'-0" SLAB

WIDTH 7'-0"   DEPTH 25"   DEPTH TO STEEL $22\frac{1}{2}$"

STEEL $\frac{7}{8}$" □ BARS 3" C. TO C.




$v = \frac{18880}{12 \times 22.5} = 70$

Shear taken by steel = S

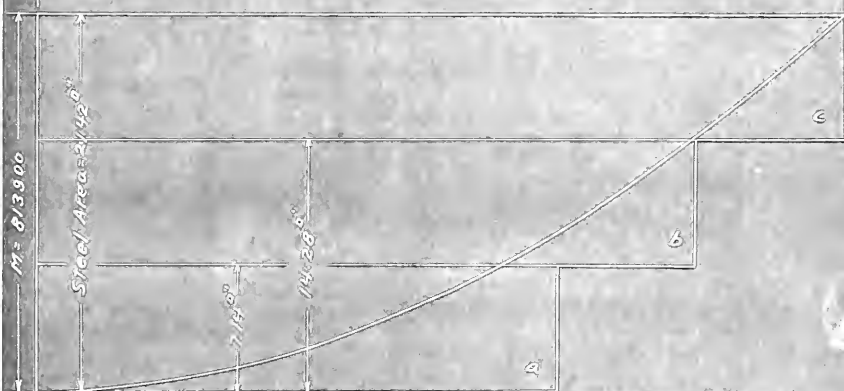
$S = 30 \div 2 \times 84 \times 40.8 = 51500^{\#}$

Alternate rods a, b, + c.

Consider "a" as taking entire stress

51,500 ÷ 7.14 = 7230#/sq. in stress in a.

SHEAR DIAGRAM

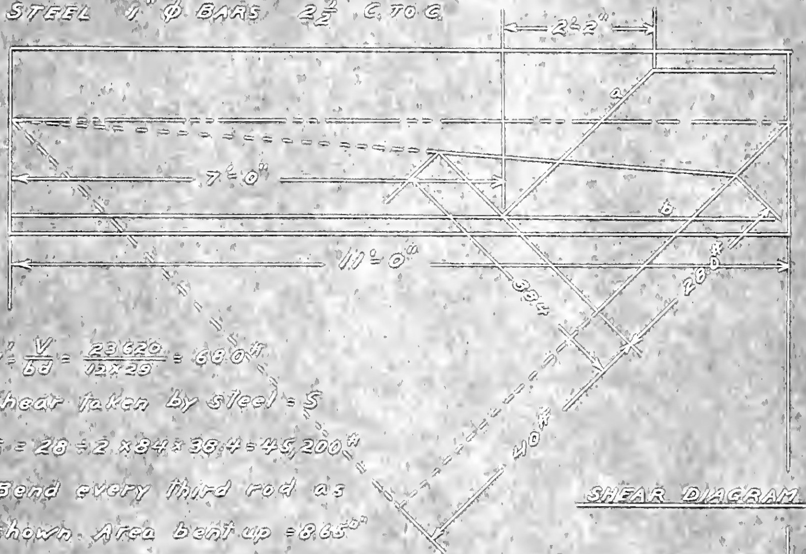



MOMENT DIAGRAM

# 22'-0" SLAB

WIDTH 7'-0" DEPTH 32" DEPTH TO STEEL 29"

STEEL 1" $\phi$ BARS $2\frac{1}{2}$" C TO C




$v' = \frac{V}{bd} = \frac{23620}{12 \times 29} = 68.0\#$

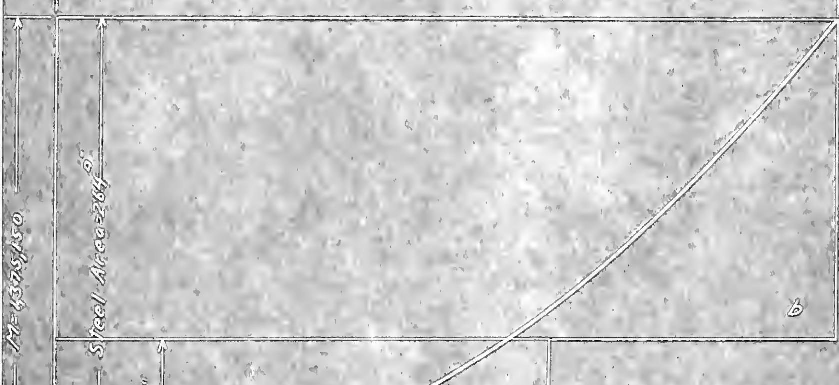
Shear taken by steel = S

$S = 28 \div 2 \times 84 \times 38.4 = 45{,}200\#$

Bend every third rod as shown. Area bent up = 8.65□"

$45{,}200 \div 8.65 = 5230\#$/sq in stress in steel

SHEAR DIAGRAM




MOMENT DIAGRAM

## DESIGN OF PILE BENT

The maximum live load reaction at a bent supporting two 17 foot slabs is 147,200#. The dead load reaction is 390x17x14=92,500# The total reaction=147,200+92,500 = 239,700# While this is the reaction for seventeen foot spans it will be used in designing the bents for all spans.

In the absence of any knowledge of the soil conditions at the trestle a pile will be considered as carrying a load of only twenty tons. This value is low enough to allow for the most unfavorable conditions.

$$239,700 \div 40,000 = 5.99$$

Therefore 6 piles will be used in each bent, each one fourteen inches square.

Area of pile = 196 sq. in.

$\frac{40,000}{196}$ = 204# per sq. in. compression of pile. This value is well within the required limit.

In designing the pile cap we will assume that one pile has settled or failed, causing the cap to act as a continuous beam over a span of four feet six inches (see blue print)

Live load reaction = 147,200#

Dead " " = 92,500#

Total " " = 239,700#

Reaction per foot = 17,100#

Weight of cap per foot = 1,000#

Total weight " " = 18,100#

$$M = \frac{wl^2}{12} = \frac{18,100 \times 54^2}{12} = 4,400,000 \text{ in. lbs.}$$

k=.4[illegible] j=.8[illegible] [illegible]

$$As=\frac{[illegible],[illegible],[illegible]}{1[illegible]000.[illegible]x.[illegible]}=11.00[illegible] [illegible].$$

$$fc=\frac{2[illegible],[illegible],000}{1[illegible]000x.[illegible]x.[illegible]x[illegible]^2}=[illegible]0\#$$

This [illegible] [illegible] be reduced to 750# by using compressive steel.

Area of tensile steel=1.0[illegible]

Allowed [illegible] stress=750#

Per cent reduction $\frac{[illegible]0-750}{[illegible]0}$ = 1[illegible].[illegible]

From table 12 Turneaure and Maurer we find that .[illegible] of compressive steel is needed [illegible] [illegible] [illegible].

.00[illegible]x[illegible]x[illegible]=1.[illegible] [illegible]. in.

Use [illegible]--I in. square bars in bottom of [illegible].

Use 5--I in. " " at top " "

Maximum shear=1[illegible],[illegible]x[illegible].[illegible]=[illegible],[illegible]00

$\frac{[illegible],[illegible]00}{[illegible]}$ =[illegible].0# shear

Since this value is within the allowed value for concrete, no steel is needed to resist the shear.

Double pile caps will be used for every fifth bent. About 1 1/2 times as [illegible] steel will be used as in the other caps.

## DESIGN OF PIER

When the distance from base of rail to the ground exceeds sixteen feet, thin piers will be used instead of pile bents. For slabs with a twenty-two foot span, piers will be used in all cases. The pier will be designed for a twenty two foot span and the same section used for all other spans. The stresses that the pier must resis consist of direct compression, due to the weight of the structure and load, and bending moment, due to the stopping of a train on the trestle. The use of an expansion joint at each pier eliminates bending due to the expansion and contraction of the slabs, and the fact that thin piers are used reduces the wind and current stresses to a negligible quantity.

Weight of 22ft. span =116,500#

" " pier about 67,000#

Maximum pier reaction=175,400#

Total load on pier =358,900#

Pier cross section =24 in. x 168 in.

358,900÷40,000=9 piles needed.

358,900÷(24x169)=130# compression.

Maximum load in one panel due to uniform train load = 22x5000=110,000#

Force acting at top of rail due to stopping of train= 1/5x110,000=22,000#

Since the height of pier [illegible] will assume an extreme case in designing the steel to resist bending. Assume a height of twenty feet to rail top.

M=[illegible]000x20x12=5,27[illegible],000 in. lbs.

$$A=\frac{5,[illegible],000}{16000x.[illegible]}=1[illegible] \text{ sq. in.}$$

$$Fc=\frac{2x5,[illegible],000}{.[illegible]x.857x14[illegible]}=5[illegible]\#$$

Use 20 one inch square bars spaced uniformly across the pier.

PIER FOOTING

Assume a depth of footing of [illegible] inches and a depth to steel of 32 inches.

Area of section resisting shear=

2x[illegible]=10,740 sq. in.

353,900÷10740=33.4# per sq. in.

$$M=\frac{353,900}{2} \times 18=3,[illegible],000 \text{ in. lbs.}$$

$$A=\frac{3,[illegible],000}{16000x.877x[illegible]}=7.0[illegible] \text{ sq. in.}$$

Use 14-3/4 in. square bars.

$$Fc=\frac{2x3,[illegible],000}{.[illegible]}=10[illegible]\#$$

# DESIGN OF ABUTMENT

The abutment shown on plate 1 will be used in all cases. We will consider a surcharge of five feet and figure the earth pressure at the various depths. The stability of the abutment is determined on plate 1 using the following figures for earth pressures and weights.

p=.3x120x1[illegible].33=5[illegible]

p=.3x120x8.[illegible]0 =317#

$P = \frac{317\ 533}{2} \times 12.5 = [illegible]000\#$

This pressure P acts at a distance of 11 feet from the bottom of the footing.

Wt. of 1 ft. of front wall=[illegible]340#

" " " " " earth [illegible] =[illegible]080#

" " 2 side walls =45000#

" " 2 " " per ft.=

of abutment face =[illegible]#

Dead load reaction from [illegible] slab
=[illegible]00#

All these forces are shown acting at their respective points of application in plate 1. They are combined graphically and the resultant R is found to fall within the middle third.

The vertical component of this result R is 14[illegible]#

Total dead load on abutment =14750x14=206,000#

Live load reaction=132,000#

Total load on abutment piles=338,000#

338,000 ÷ 40,000=10 nearly.

Use 10 piles in abutment.

Con[illegible] part of [illegible] wall [illegible] 1.7' of lines BB (see diagram) to be supported by the diaphragm.

Pressure at top [illegible] 120 [illegible] 110#

" " bottom [illegible] 17.5 [illegible]

Total pressure = [illegible] (6x3.75 [illegible]) = [illegible]0000#

Total tension [illegible] =18000#

45000 ÷ 18000 = [illegible].50 sq. in. needed.

Use 14-1/2 in. rods.

Consider the part of wall between the diaphragm and the front wall to act as a single [illegible] horizontal earth pressures. At the bottom the pressure per square foot is 216#

$M = \frac{w\,l^2}{8} = \frac{216 \times [illegible]}{8} = [illegible],700$ in. lbs.

d = 18-3 = 15 in.

$A = \frac{[illegible]0700}{15000 \times .[illegible] \times [illegible]} = .107$ sq. in. per ft.

$Fc = \frac{2 \times [illegible]0700}{.428 \times .[illegible] \times 12 \times 1[illegible]} = 427\#$

Use 1/2" square bars [illegible] 12 in. [illegible].

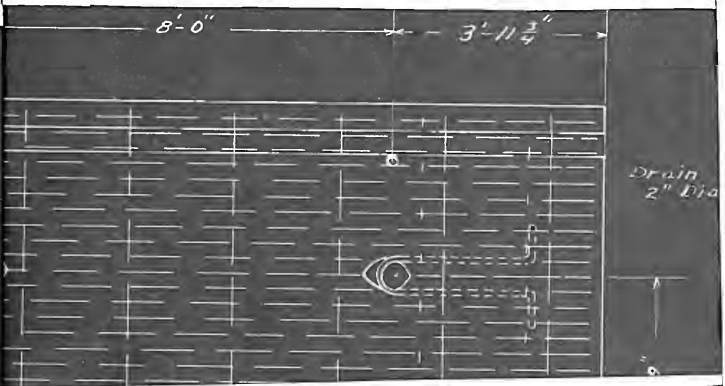
The pressure at the bottom of the [illegible] wall [illegible] 432# per square foot.

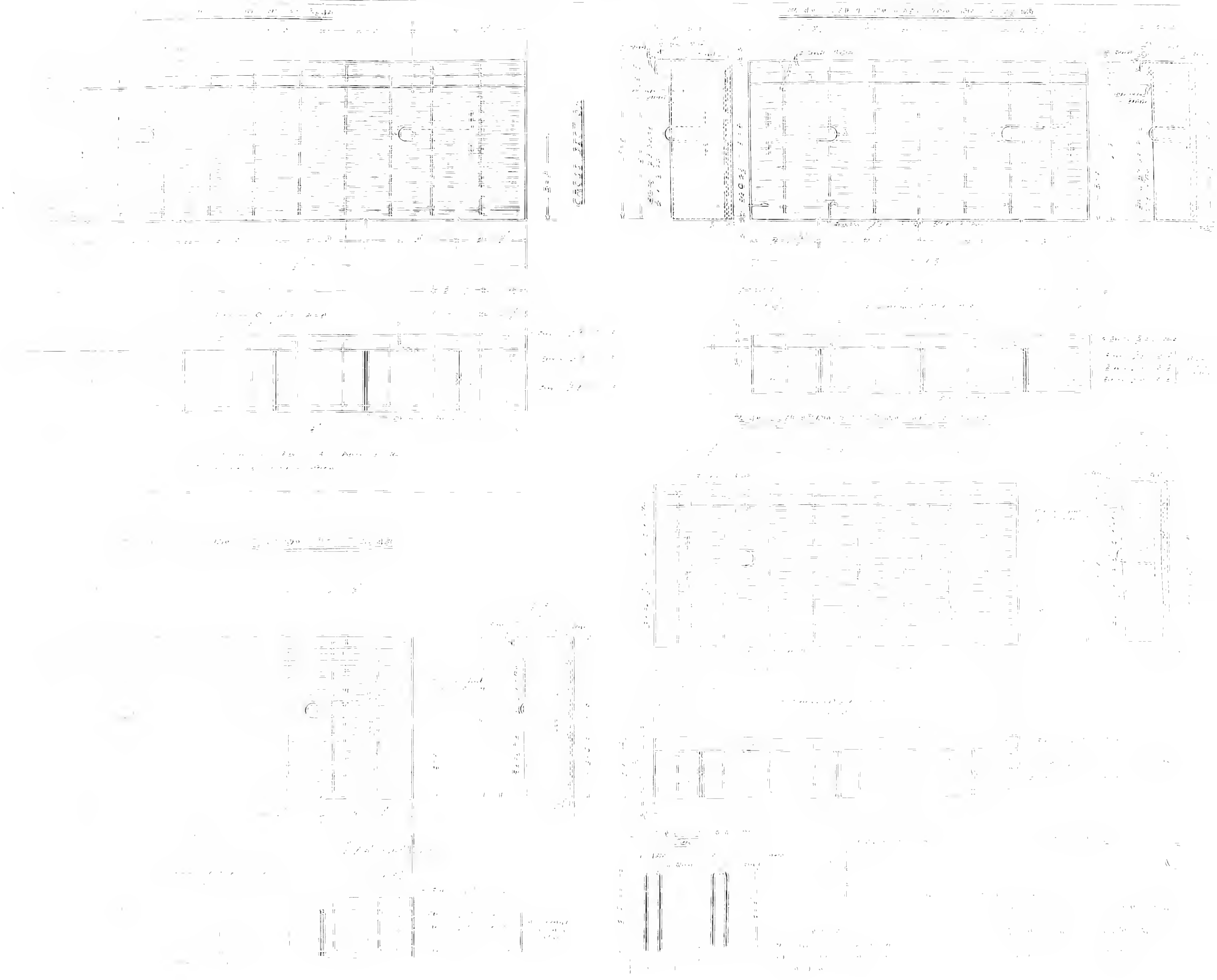
$M = \frac{w\,l^2}{8} = \frac{[illegible] \times 14^2 \times 12}{8} = 1[illegible]7000$ in. lbs.

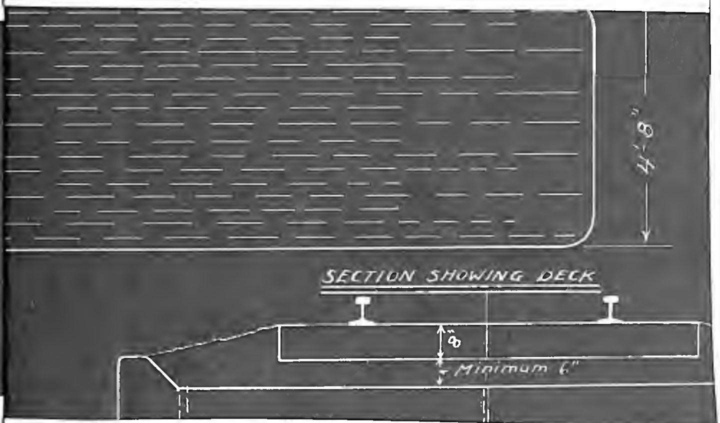
$A = \frac{2 \times 127000}{.428 \times .357 \times 12 \times 21^2} = 1[illegible]1\#$ per sq. in.

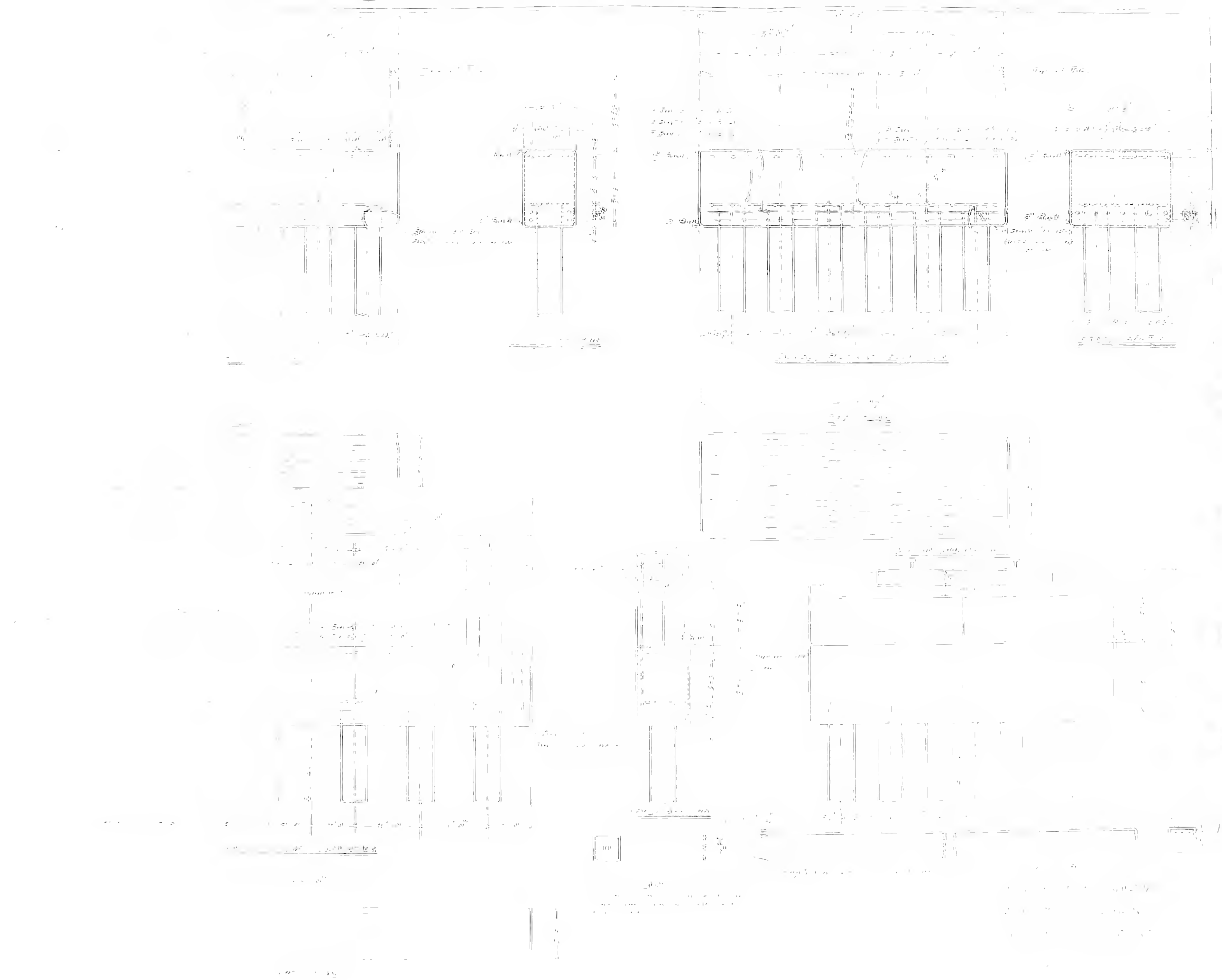
Use 3/4" square bars at 14 inch [illegible].

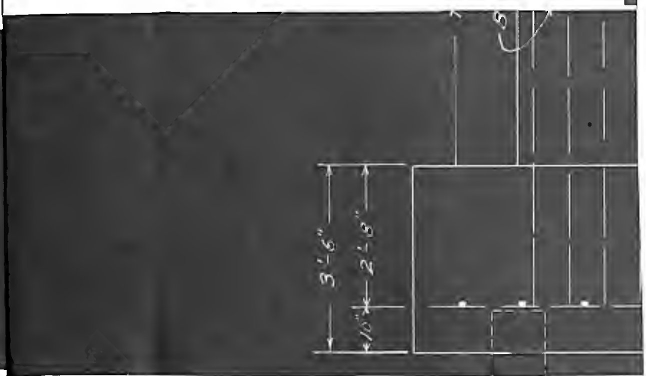
In addition to the [illegible] figured [illegible] are used for the purpose of holding the [illegible] in place as well as to take [illegible] of small [illegible] stresses.

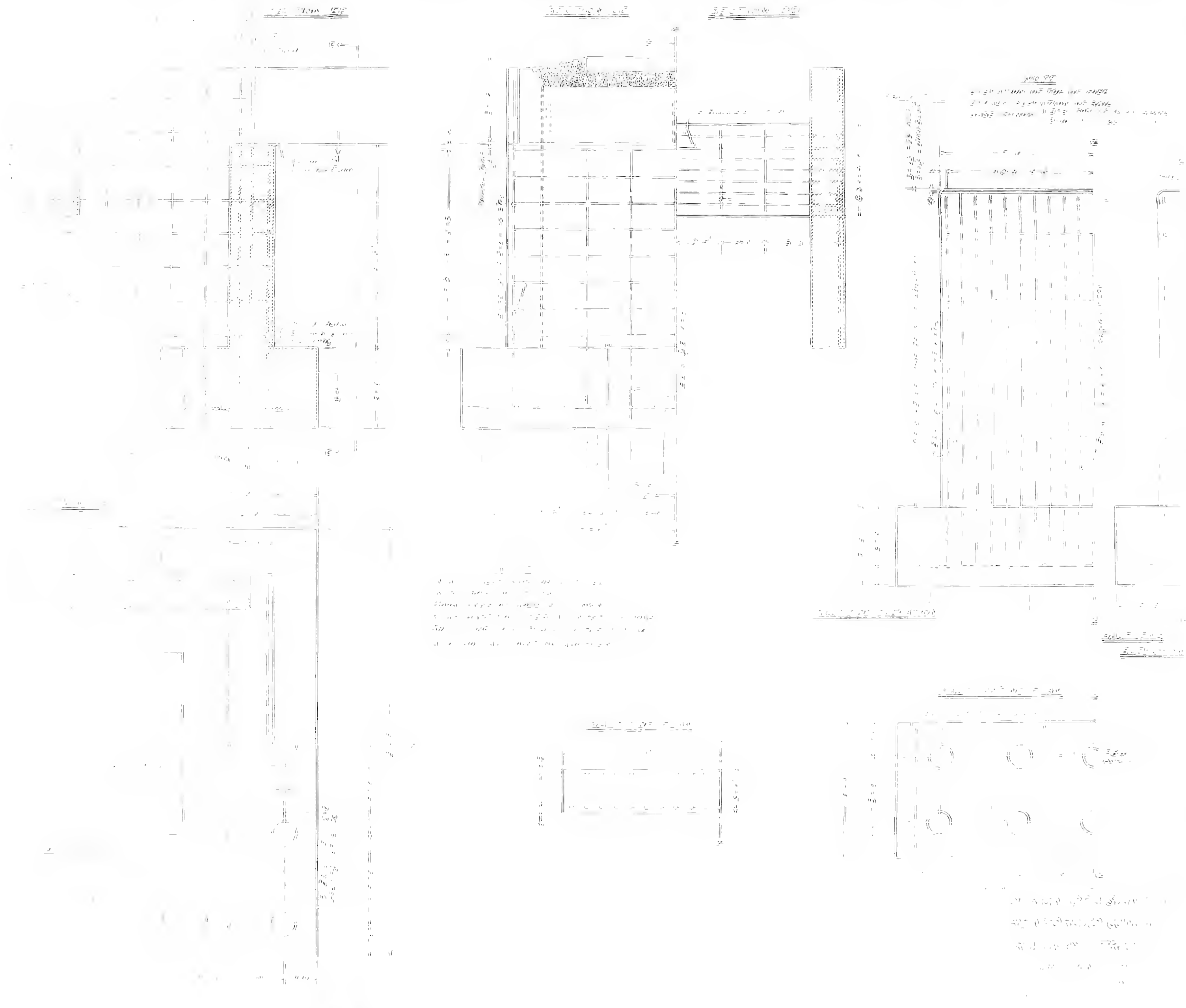

8'-0"
3'-11 3/4"
Drain
2" Dia




4'-8"
SECTION SHOWING DECK
8"
Minimum 6"




3'-6"
2'-8"
10"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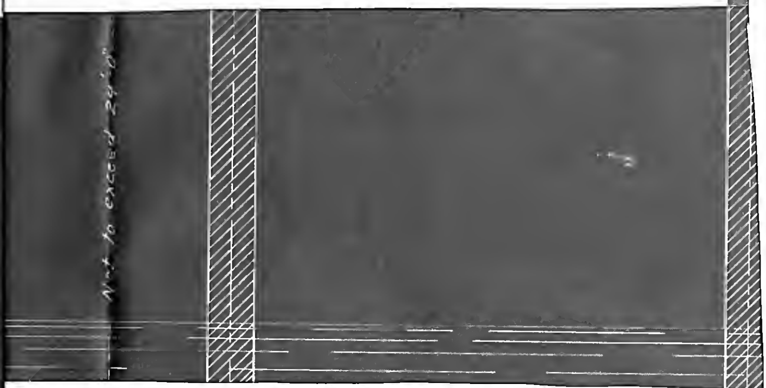
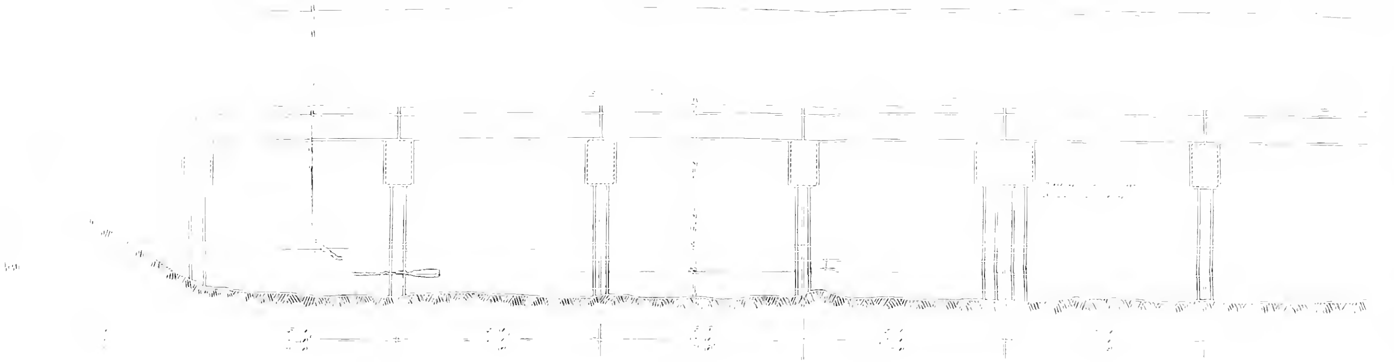

Not to exceed 24'-0"



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